

Semi-active management of structures subjected to high frequency ground excitation

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ABSTRACT: The structural response to high frequency ground excitations resulting from blast loading differs significantly from that of low frequency excitations such as earthquakes. Due to local mode vibrations in the structure the response is more complex with the maximum response generally occurring after the excitation has ceased. As a result, most of the motion takes place in the free vibration phase, which is dominated by low frequency modes with large displacements and small accelerations. However, the response in the forced vibration phase involves localised high frequency modes with small displacement and large accelerations, thus inducing a large shear force. Therefore, even if a structure is strong enough to survive the initial ground excitation, it may still be damaged in the significantly long free vibration phase. Non-linear structural analyses were carried out on a two storey reinforced concrete frame subjected to high frequency ground excitations. Several semi-active devices and device architectures were modelled, including passive tendon based solutions. The use of semi-active devices significantly reduces the free vibration response of the structure, thus reducing the extent of the structural damage generated and the likelihood of structural failure. In particular, the combination of these semi-active devices in a specialised 2-4 implementation with a yielding tendon fuse-bar provides protection against the initial pulse and the resulting free vibration without increasing shear forces. Thus it creates a solution suitable for both new and retrofit applications.

1 INTRODUCTION

Damage from blast loads can occur in two separate stages. Firstly, the initial blast impacting on the structure may cause the structure to fail due to large shear stresses generated by higher order modal responses. This failure occurs when structures are in the direct vicinity of the blast, or are the subject of the blast. As this situation directly depends on the size and proximity of the blast load, the structures usually cannot be saved by use of various devices. The second method by which structures can fail due to blast loads occurs after the initial impact, in the structures free vibration response to the blast. Structures can literally shake themselves to pieces. This second failure mode usually occurs in structures that are near by blast loads, but not directly affected by the full force of the initial blast. Structures can survive the initial blast, but the free vibration response induced by the blast can cause the structural failure due to the displacement of the structure (Dhakal, R.P. et al 2003). If this displacement can be reduced, the likelihood of failure is also reduced.

An ideal method of reducing a structures dis-

placement during its free vibration period is to use semi-active devices. Resetable devices non-linearly alter the stiffness, rather than the damping, of the structure with the stored energy being released as the compressed fluid reverts to its initial pressure. These emerging devices have been studied extensively for seismic structural control (Barroso, LR et al 2003) and have the ability to re-shape hysteretic behaviour in some implementations (Mulligan et al 2005).

The amount of displacement in the free vibration phase of the structures response depends heavily on the size of first displacement peak resulting from the blast load. Hence a tendon is used to reduce the magnitude of the initial displacement, that is, the first displacement peak. A resetable device is used to reduce the subsequent free vibration displacement, reducing the size of the free vibration phase. The tendon and the resetable device acting in tandem will therefore reduce the likelihood of failure due to displacement.

2 TENDON

2.1 Introduction

The initial pulse of a blast that affects a structure cannot be avoided. Generally, if the blast is large enough, a structure will fail. For smaller blast loads that cause the structure to fail later due to the induced vibrations, the damage can be mitigated. If the distance which the structure displaces is reduced, the resulting vibrations will also be reduced, thus reducing the likelihood of structural failure is the free vibration phase. The trade off is that introducing a tendon will increase the shear stress in the structure increasing the likelihood of failure due to shear stresses. To avoid such failure, the tendon is set to yield before the shear stress reach critical levels. Tendons with fixed level yield-based fuses offer a means of reducing the initial displacement without exceeding a safe level of shear stress.

2.2 Tendon properties

It is not desirable for a tendon to be acting the entire time a structure is undergoing vibration because the shear stresses in the structure will increase, increasing the amount of damage done to the structure. Ideally, a tendon would only act during the forced vibration stage of a blast loaded structural response. To maximize the effectiveness of a tendon without significantly increasing the shear stress on a structure, the tendon is set up with a fuse bar so that the bar will yield and break before the first peak displacement is reached.

Ideally, the bar would yield at the maximum displacement of the structure. In typical blast loaded structures this peak is the first peak of the response to a blast load. The bar would therefore have to be set up so that it would not yield too early and have little effect on the response. However, too high a yield point and it would not yield at the peak displacement, continuing to act throughout the entire response. As the size of the blast load cannot be predicted however, the tendon cannot be set up to yield at the ideal yield point. Instead the tendon is set up to yield at a point which will reduce the subsequent displacement, but not increase the shear stress in the system to detrimental levels.

2.3 Tendon layout

The tendon layout should be such that it resists the overturning moment of the lateral loads on the structure (Pekcan et al., 2000). For this research a single draped tendon is used. However, in practice

two would be used, one tendon on each side of the structure, because the direction from which the blast load excites the building would be unknown. Hence, it allows for blast loads from both directions. The tendon layout analysed drapes the tendon in a shape approximately the same as the structural shear diagram (Pekcan et al., 2000), as shown in Figure 1.

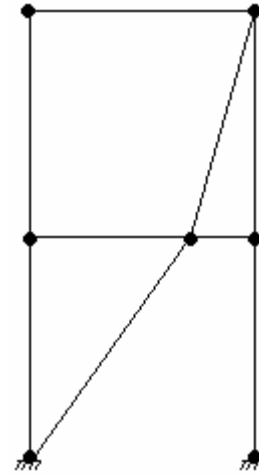


Figure 1 Tendon layout schematic

3 SEMI-ACTIVE DEVICE

3.1 Introduction

Semi-active control is an effective method of mitigating structural damage from large environmental loads. Semi-active systems do not add energy to the system, guaranteeing stability. Semi-active devices are capable of staying dormant for long periods of time until they are suddenly required to produce large forces. Instead of altering the damping of the system, semi-active devices non-linearly alter the stiffness of the system. Constructed to utilise well understood fluids such as air, semi-active devices are reliable and simple. Semi-active devices can be easily added to existing structures or incorporated in new structures, making them an ideal answer to vibration mitigation.

3.2 Application

The type of semi-active device utilised in this work was a 2-4 device. The 2-4 device provides significant damping in quadrants 2 and 4 of the hysteresis loop. In this case only motion towards the zero position from peak values is resisted. As such, the 2-4 device dampens lateral forces without increasing the base shear of the system (Mulligan et al 2005, 2006).

The size or effective added stiffness of the device can be varied as can its position on the structure. Different positions and sizes alter the structural response to varying degrees. Through numerical simu-

lation the optimal size and position of a 2-4 device can be found for a generic structure undergoing a blast load.

Overall, the 2-4 device resists only motion towards equilibrium. The tendons only resist the first peak motion away from equilibrium. Hence, the optimal location of the tendon and 2-4 device can be determined independently in the design of the overall system.

4 STRUCTURE AND NUMERICAL MODEL

A simple two storey structure was modelled numerically. The layout of the two storey RC frame and its geometric details are shown in Figure 2. The frame was 3.6 metres wide with a total height of 7.2 metres. The columns of the structure were 0.3 metres thick reinforced concrete with an elastic modulus of 10×10^9 Pa. The structure is designed so that it has a natural frequency of 0.7Hz. Each floor is assumed to weigh 1000kg with a live load of 630kN acting on the second floor and 450kN on the first floor.

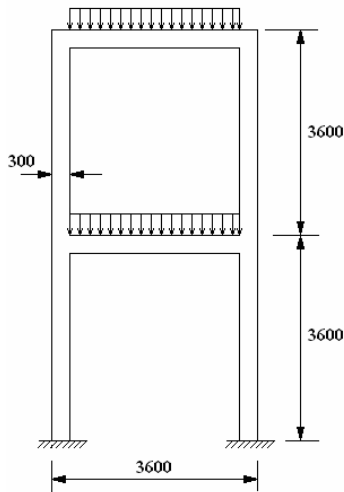


Figure 2 Target two-storey RC frame

Because of the nature of blast loads, a traditional finite element structure with the column masses lumped at the floors is not desirable. Such a model will not pick up the higher frequency responses of the structure that are present in blast loads. To correctly model the structures behaviour the columns mass must be distributed through out the column. It has been found that discretising the column mass in 6 stages through out the column is sufficient to accurately model the structural response (Lu, Yong et al. 2001). These higher frequencies are important as the initial short pulse load can excite them, resulting in initial damage facilitating failure in the later free response.

The structure was modelled using nonlinear finite elements employing a bilinear hysteretic model. These elements were used so that yielding of the

structure could be accurately modelled and captured.

The blast wave is modelled as a simple triangle wave with a peak of 35kN and a width of 0.05 seconds. It is assumed that the peak is reached instantaneously, after which the blast load reduces linearly to zero over the time period of 0.05 seconds. This model is typical of blast loads, which exert large accelerations over small time scales. Such a blast load can be expected to excite higher order modes in the structure, and a second order modal response would not be unexpected. To retain the triangular shape of the blast load as much as possible a smaller time step is used while the blast load takes place. After this initial period a longer time step is suitable.

5 RESULTS

The two storey RC frame was subjected to a blast wave of with a peak pressure of 35kN and the time-history dynamic response calculated. The displacement, velocity and acceleration histories for each floor in the frame were obtained. The frame was subjected to the blast wave with no control devices, with only a tendon, and with a tendon and a semi-active 2-4 device. The tendon size and location was varied for each run. The semi active device size and location was also changed for each simulation, with its location being varied between the first and second floors and the two columns. This parametric analysis was performed to determine the configuration that gave the maximum reduction in displacement.

Table 1 shows the effect of varying the size of the 2-4 device attached to the first floor and the ground. The parameters measured are $\|Y\|$, the energy of the system, F11, the first peak displacement of the first floor, F12, the difference between the second peak displacement of the second floor and its first peak displacement, F21, the difference between the second peak displacement of the first floor and its first peak displacement, and F22 the second peak displacement of the second floor. F12 and F22 are presented as absolute values, as they occur in the negative portion of the response. Finally, all values were normalised to the uncontrolled case.

Table 1 Effect of increasing the 2-4 device stiffness normalised to the uncontrolled case (k is the nominal column stiffness).

Parameter	uncontrolled	tendon		
		only	0.01k	0.05k
$\ Y\ $	1.000	0.207	0.207	0.209
F12	1.000	0.787	0.766	0.690
F11	1.000	0.765	0.765	0.765
F22	1.000	0.263	0.261	0.269
F21	1.000	0.436	0.439	0.452
Parameter	0.1k	0.5k	k	2k

$\ Y\ $	0.200	0.211	0.340	0.399
F12	0.616	0.347	0.231	0.194
F11	0.765	0.765	0.765	0.765
F22	0.284	0.265	0.219	0.188
F21	0.468	0.535	0.569	0.598

The effect of the tendon is significant. The peak values F11 and F21 are reduced by between 29% to 56%, indicating the effect of the tendon only on this first cycle. The 2-4 device was then added and its size gradually increased as percentage of the column stiffness k . The 2-4 device had little effect until it was at least 10% of the column stiffness (0.1k) in size. It has no effect on F11 and F21 as it only begins to act after that first peak. With the 2-4 device at 2k the total energy in the system, $\|Y\|$, is reduced by 14 percent more than the tendon alone, and the response thus has fewer damaging cycles.

Table 2 is similar to Table 1 except the 2-4 device is applied between the second floor and the ground. In this case the 2-4 device is much more effective. When the device has a stiffness of 2k the structure becomes overdamped, and only one peak occurs so no F12 and F22 are observed.

Table 2 Effect of increasing the 2-4 device stiffness normalised to the uncontrolled case (k is the nominal stiffness).

Parameter	Uncontrolled	tendon		
		only	0.01k	0.05k
$\ Y\ $	1.000	0.207	0.176	0.142
F12	1.000	0.765	0.765	0.765
F11	1.000	0.765	0.765	0.765
F22	1.000	0.263	0.230	0.165
F21	1.000	0.436	0.436	0.436

Parameter	0.1k	0.5k	k	2k
$\ Y\ $	0.131	0.178	0.198	0.246
F12	0.765	0.765	0.765	0.765
F11	0.765	0.765	0.765	0.765
F22	0.126	0.000	0.000	0.000
F21	0.436	0.436	0.436	0.436

Figure 3 shows the time history of the uncontrolled structure. The main concern is the time in which the displacements take to decay down to zero or the energy of the response. Other points of note are the second modal response of the first storey and the amplitude of the first peak, as seen in the first peak of storey 1. Finally, the initial, large peak responses are a measure of potential damage, as well.

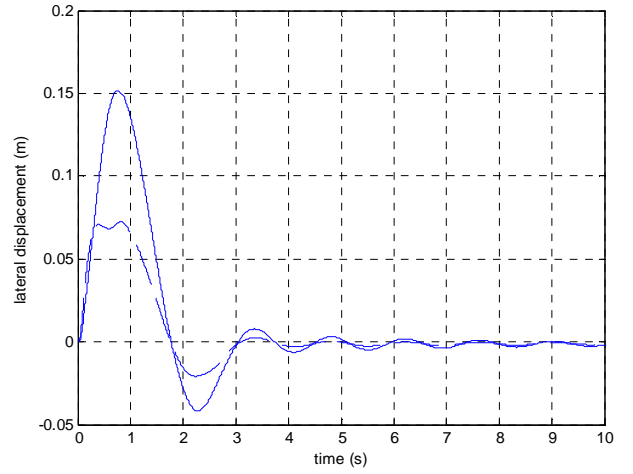


Figure 3 The uncontrolled structural displacement

Figures 4 and 5 show the effect of a 2-4 device at 10% of the structures column stiffness (0.1k) coupled with a tendon at different floors. Both figures show the use of a tendon and a 2-4 device significantly dampen the displacement resulting from a blast load. Figure 5 shows that the device placed between the second floor and the ground is more effective than Figure 4 where the 2-4 device is placed between the first floor and the ground. This result is likely due to the device offering greater resistance to the overturning moment when placed on the second floor.

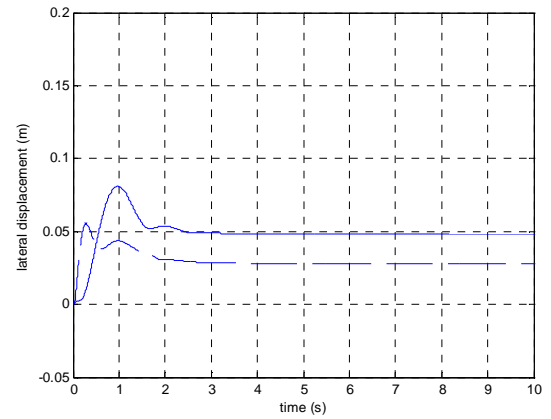


Figure 4 Structural response with the 2-4 device on the first floor and tendon

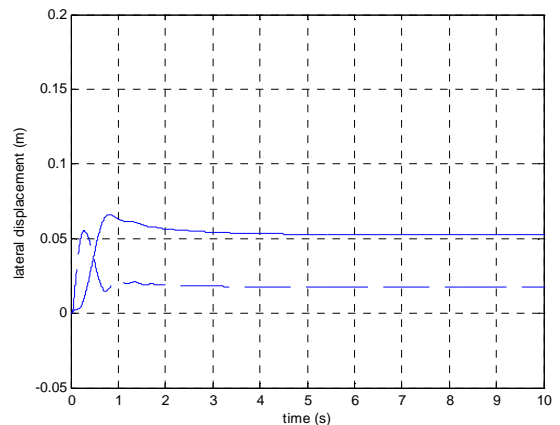


Figure 5 Structural response with the 2-4 device on the second floor and tendon

The metrics studied cover all major damage related aspects of the response. The first peaks (F11, F21), are related to the initial blast damage and can exhibit higher mode behaviour also. F12 and F22 indicate how the tendon and initial semi-active device can further reduce damaging peak values. Finally, $\|Y\|$ measures the total dissipation by the system, focusing mainly on the semi-active device, as it accounts for the entire response.

Therefore the overall results for all these metrics indicate the tendon and device provide a synergistic effect. The tendon mutes the initial damaging displacement, setting up a shorter and smaller free response. The 2-4 device minimises response energy without increasing base shear loads on the foundation due to its unique implementation. Hence, in combination all major metrics are reduced significantly without significant added shear loading – a particular benefit for retrofit applications.

6 CONCLUSIONS

Nonlinear finite element modelling of a two storey structure with distributed column mass effectively models the response of a structure to a blast load. The main part of the displacement of the structure occurs after the blast has affected the structure. The free vibration phase is where the displacement of the structure is likely to cause damage.

Through the use of a 2-4 device positioned appropriately on a structure in conjunction with a fuse-tendon system the displacement resulting from a blast load can be effectively damped so that the structure has a high probability of surviving the blast. The system can be easily applied to new and existing structures due to its inherent simplicity.

The ideal size of the 2-4 device is within 10% to 100% of the structures column stiffness, placed between the second and ground floors of the structure. This dampens out the displacement reducing the free vibration period and thus reducing the likelihood of failure due to displacement.

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